Effect of Shear Wall on Seismic Performance of RC Open Ground Storey Frame Building

Ashwani Singh

Department of Civil Engineering
National Institute of Technology Rourkela
Rourkela – 769008, Odisha, India

Effect of Shear Wall on Seismic Performance of RC Open Ground Storey Frame Building

A thesis

submitted in

May 2015

to the Department of

Civil Engineering

of

National Institute of Technology Rourkela

In partial fulfillment of the requirements

for the degree of

Master of Technology

in

Structural Engineering

by

Ashwani Singh

710CE2017

under the supervision of

Prof. Robin Davis P

Department of Civil Engineering

National Institute of Technology Rourkela

Rourkela – 769008, Odisha, India

Dedicated to My Parents

Smt. Anita Singh

Shri Sube Ram Singh

National Institute of Technology Rourkela
Rourkela - 769008, Odisha, India

Certificate

This is to certify that the work in the thesis entitled ***"Effect of Shear Wall on Seismic Performance of RC Open Ground Storey Frame Building"*** submitted by ***Ashwani Singh***, having roll number ***710CE2017***, is a record of an original research work completed by him under my guidance and supervision in partial fulfillment of the requirements for the award of the degree of ***Master of Technology*** in ***Structural Engineering, Department of Civil Engineering*** . Neither the content of this thesis, nor in full or any part of it has been submitted to any other Institute or University for any degree or academic award.

Prof. Robin Davis P
Assistant Professor
Department of Civil Engineering
National Institute of Technology Rourkela

Acknowledgement

First and foremost, I thank God, The Almighty, for his grace and blessings on me throughout the entire journey to pursue this research work successfully.

My heartily gratitude to my beloved guide **Prof. Robin Davis P, (Dept. of Civil Engineering, NIT, Rourkela)** for enlightening me with his immense knowledge and for his patience, motivation, spontaneous guidance, precious time and valuable suggestions. His encouragement and guidance supported me always during my research work as well as thesis writing.

Besides my mentor i extend my special thanks to *Prof. S. K. Sahu*, the Head of the Civil Department, *Prof. Pradip Sarkar*, *Prof. Manoranjan Barik* and all other faculties of structural engineering department for their constant encouragement and support.

My warmly thanks to all my companions for their positive feedback, adoring cooperation and brilliant advice during this journey. My special thanks to *Haran Pragalnath D* and *Manas Ranjan Pradhan* for their help in thesis writing.

Finally, wholehearted regards to my mother *Smt. Anita Singh* and my father *Shri Sube Ram Singh*, for their endless love and support. I have no words to express my gratitude towards my family for sacrificing so many things for the betterment of my life.

Ashwani Singh
710CE2017
Structural Engineering

Abstract

***Keywords*:** Open Ground Storey Building, Reinforced concrete shear wall, Multiplication Factor, Linear Static Analysis, Nonlinear Static Analysis (Pushover Analysis).

The Open Ground Storey buildings are very commonly found in India due to provision for very much needed parking space in urban areas. However, seismic performance of this type of buildings is found to be consistently poor as demonstrated by the past earthquakes. Some of the literatures indicate that use of shear walls may enhance the performance of this kind of buildings without obstructing the free movement of vehicles in the parking lot. The present study is an attempt in this direction to study the performance of Open Ground Storey buildings strengthened with shear walls in a bay or two. In addition to that, the study considers a different scenarios of Open Ground storey buildings strengthened by applying various schemes of multiplication factors in line with the approach proposed by IS 1893 (2002) for the comparison purpose. Study shows that the shear walls significantly increases the base shear capacity of OGS buildings however the comparative cost is slightly on the higher side.

Contents

List of Figures

List of Tables

Chapter 1

Inroduction

1.1 Seismic Behaviour of Open Ground Storey Buildings

The idea of open ground storey (OGS) building has been introduced mainly because of the need for parking in urban localities. Due to the special feature of providing parking facility in the ground storey of this building, a large number of open ground storey buildings have been built and accommodated especially for residential purposes throughout the different cities of the country. In actual sense, when the columns of a reinforced concrete building are left open without providing any masonry infill wall as partition wall in between them to have parking area in the ground storey then this type of structure can be treated as open ground storey or soft storey building. The most important issue can be verified that the ground storey is quite flexible in nature in comparisons to the other upper storeys of this building. This literally means that the relative storey drift of the ground storey is quite larger with respect to the other upper storeys of such buildings when subjected to earthquake loads. Consequently, the ground storey is exceptionally weak against other upper storeys to resist large earthquake forces usually present at the ground storey of the building.

Throughout the world, open ground storey buildings consistently performed poorly under many earthquakes which happened in the recent years because of the irregularity of stiffness and strength in the ground storey and upper storeys of this building and a huge number of them got collapsed easily. In past years, a large number of open ground storey buildings had been constructed in different parts of the India like in Ahemdabad, one of the main cities of India consists of mainly around 25,000 five-storey buildings and about 15,00 eleven-storey buildings. Basically, most of them are open ground storey buildings. In addition to that, a huge construction of open ground storey is going on to build high-rise residential buildings having this feature i.e open ground storey and already exists in different towns and cities of the country located in moderate to highly seismic active areas as per Indian Standard.

The study after Bhuj earthquake happened in 2001 at Ahemdabad has explicitly mentioned that open ground storey building is unsafe and highly vulnerable to earthquake shaking. Due to the presence of masonry infill wall in upper storeys made them much stiffer than the ground storey. Therefore, it creates difference of stiffness between the ground storey and upper storeys of open ground storey building. Thus, the horizontal drift of ground storey is quite large relatively and the upper storeys of this building displaces like a single block. Subsequently, if the columns of ground storey are not strong enough to resist large horizontal loads like earthquake forces and are not provided adequate ductility then they may get highly damaged which may lead to the catastrophic collapse of such buildings.

1.1.1 Motivation of the study

Open ground storey building is inherently poor structure with abrupt change in stiffness and strength at the ground storey level. The problem occurred because of neglecting the presence of masonry infill wall and only bare frame elements are considered while designing open ground storey building. Hence, the effect of inverted pendulum has not been taken into account. Many improved and important design provisions and guidelines have been formulated in Indian

Standard IS 1893 (2002) regarding open ground storey buildings after studying the case of Bhuj earthquake occurred in 2001.

In the beginning, it defines explicitly when a structure is termed as a weak or soft storey building. Next, it recommends to consider higher amount of design forces in comparison to the other storeys while designing soft storey of such building. The code also suggested to calculate design forces of the bare frame building without considering the effect of masonry infill under earthquake loading and then design the columns of the ground storey of an open ground storey building by applying a factor, 2.5 times the forces calculated from the analysis of bare frame building. The factor in which the columns of the Open Ground Storey building to be multiplied is termed as multiplication factor (MF) in this study. A number of studies [2] suggest to provide walls in the ground storey made up of either of masonry or reinforced concrete wall (shear wall) to avoid the unfair and irregular distribution of stiffness and strength in any storey of the building. The present study attempts to compare the seismic performance of OGS frame strengthened with RC shear wall. The study includes the OGS frames designed with various schemes of MF in the columns of ground and first storeys.

1.2 Reinforced Concrete Shear Wall

1.2.1 Theoretical Background

It would be correct to say that there are as many kinds of lateral resisting systems as there are intellectual humans like engineers, scientist etc. Basically, most of them are divided into three sections.

1. Reinforced concrete frame system
2. Shear wall system
3. Dual system, the Shear wall – frame system

From the engineering point of view, the most preferred system for design of high-rise buildings is the shear wall-frame system i.e dual system. Now a days, reinforced concrete frame buildings are engineered with the application of structural walls like reinforced concrete shear walls and these buildings are performing better under seismic action in comparison to reinforced concrete frame buildings by reducing the probability of excessive deformations and hence collapse.

Generally, shear walls are normally constructed at the foundation level and are continuous following the height of the building. The provision of thickness starts at minimum value of 150 mm and ends at maximum of 400 mm in high-rise structures. These structural walls are usually provided in both directions of the building. Shear walls support gravity loads and simultaneously resist lateral loads by diaphragm action and transfer them to the foundation. Lateral or horizontal forces applied to the building are derived from earthquakes result shear and overturning moments in shear walls. The shear force tends to tear up the shear walls in various parts. The tendency of the shear wall to be lifted up at one end where lateral load is applied and to be pushed down at the other end resists the overturning moment produced due to earthquake loads.

The maximum amount of lateral or horizontal shear force is completely resisted by shear walls due to this action, these structural walls are named as shear walls. The capability of shear walls to resist lateral storey shear forces, storey torsion and overturning moments primarily based on its location, orientation and geometric configuration within the structure [3].

1.2.2 Shear wall – frame Interaction

This system has a combination of shear wall and RC frame provides a resistance to lateral loading. The potential of wall-frame structure totally depends on the extent of horizontal interaction, which is controlled by the relative stiffness of the reinforced concrete frames and shear walls and the height of the building. The taller the structure and the stiffer the RC frames, the larger the interaction. The RC frame is deflected in shear mode while the shear wall is responded in bending as similar to a cantilever. The structural compatibility of lateral deflection develops interaction between them.

The lateral sway of the RC frame combined with the shear wall deflected in the parabolic sway results in improved stiffness of this system significantly because the shear wall is effectively restrained by the moment frame at the top levels whereas at the bottom levels, the moment frame is restrained by the shear wall. Therefore, the combined action of structural elements is truly based on the relative rigidity of the both and their respective modes of deflection. The horizontal deflections of a RC shear wall is much more identical to a cantilever column. At the bottom, the shear wall acts relatively stiff and hence, the floor to floor deformations would be less than half the values at the top. On the top floors, the lateral deflection increases rather easily due to the cumulative effect of shear wall rotation. On the other side, RC frames adopted the shear mode of deflection. The relative deflections of the storey basically based on the value of shear force applied to the floor of each storey. However, the lateral deflection is greater at the bottom level and lesser at the top level in comparison to the shear wall, the floor to floor deformations may be considered nearly uniform in distribution following the height of the building. When the current system i.e the shear wall-frame system is connected by applying rigid diaphragm action then a nonuniform storey shear force generates between them. Consequently, the typical interaction results in a much more economical structural system [3].

1.3 Objectives of the Present Study

The broad objectives of the present study are stated herein.

1. To study the effect of providing shear walls in seismic performance of Open Ground Storey RC frame buildings.

2. To study the comparative performance of typical Open Ground Storey buildings strengthened with shear walls with reference to that of OGS buildings by applying various MF.

3. To carry out comparative cost analysis of using shear walls with reference to that of increasing the cross section of ground storey columns applying MF.

1.4 Methodology

The methodology followed to accomplish the above mentioned objectives are given below.

1. A broad literature review on the use of shear walls in the frame buildings.
2. Selection of typical four storey Open Ground Storey RC frames.
3. Linear static analysis of RC frames without considering the effect of masonry infill as per Indian Standard.
4. Designing of RC shear wall and ground storey and/or first storey columns of the four storey OGS building with various multiplication factors.
5. Modelling of the selected frame buildings to capture nonlinear behaviour.
6. Performance comparison of the buildings in terms of nonlinear static pushover curves.
7. Comparative cost analysis of each strengthening scheme.

1.5 Assumptions in the Present Study

Following are the assumptions of the present study.

1. The OGS frames are assumed to be symmetric in plan and hence plane frames are considered. Torsional effects are neglected.
2. The shear wall considered in the study is provided throughout the height of the building as studies [4] suggests the same.
3. The interaction effect of the soil-foundation structure is neglected.
4. Distributed plasticity element is employed for nonlinear modelling of RC members.

1.6 Organisation of the thesis

After an introduction of this chapter, the subsequent Chapters are presented in the following manner.

1. A literature study regarding open ground storey building combined with reinforced concrete shear wall is discussed in the second Chapter. A validation study to validate the nonlinear modelling and analysis approach is followed.

2. Third Chapter starts with description of the example OGS frames, geometry and design details and definition of various performance criteria. A discussion of linear and nonlinear static analysis of all the frames to compare their relative performances is presented subsequently. A cost analysis is discussed in the last part of this Chapter.

3. Fourth Chapter includes the major conclusions and findings from the present study.

Chapter 2

Literature Review & Validation Study

2.1 Introduction

First part of this Chapter focuses on the literature review on behaviour of OGS buildings, analytical and experimental studies on shear walls and modelling of reinforced concrete elements. The last part of this Chapter presents the validation of the nonlinear modelling and analysis approach with a published literature.

2.2 Open Ground Storey Building

Murty and Jain (2000) [5] conducted experiments on RC frames with masonry infill based on cyclic tests. It was observed that masonry infill provides significant lateral stiffness, energy dissipation capacity and ductility. With the help of some arrangement by providing reinforcement in the masonry infill, it was anchored into the column of the frame to improve effectively the out of plane response of masonry infill.

Davis *et al.* (2004) [6] studied the seismic performance of two typically existing buildings situated in moderate seismic zones of India by performing linear static analysis, response spectrum analysis and nonlinear pushover analysis. In one building irregularity in plan and vertical irregularity like soft storey were found and another building was symmetric in nature. The equivalent strut method was used to modelled infill walls.

Kaushik *et al.* (2007) [7] conducted experiments on unreinforced masonry infill for obtaining compressive stress- strain behaviour. Nonlinear stress-strain curves had been obtained for bricks, masonry, mortar six control points had been plotted on the stress-strain curves of masonry, which were used to define the performance limit states of the masonry infill.

Pujol *et al.* (2008) [8] tested a full scale three-storey structure having infill brick walls under displacement reversals. Results were compared of this test with the results of the same building without having infill walls. In the first test, at the slab-column junction, structure showed a punching shear failure. Infill walls prevented the slab collapse and effectively increased the strength and stiffness of the structure. The experimental results were calibrated to match the numerical model of the test structure. Numerical simulations suggested that the measured drift capacity was not reached even during strong motion.

Mulgund (2011) [9] designed five RC frame buildings with masonry infill walls as per IS code in order to consider the effect of masonry infill under same seismic condition because while designing of RC frame buildings usually do not consider the effect of masonry infill. The present work dealt with a study of RC frames subjected to dynamic loading with different arrangement of masonry infill walls. The results were extracted and compared for both bare frame and bare frame with infill walls. Finally, conclusions were derived and put forward in accordance of with IS code.

Prakashvel *et al.* (2012) [10] conducted a work to study the seismic bahaviour of soft storey or open ground storey building under seismic loading and their problems to make them earthquake resistant to check the catastrophic collapse of such buildings. An attempt had been made to evaluate the seismic performance of open ground storey buildings with the help of shake table test.

Sivakumar *et al.* (2013) [11] studied behaviour of the ground storey columns of multi-storey building subjected to dynamic earthquake loading. An equivalent strut method had been used for modelling of upper storey masonry infill wall panel to account the structural effect of masonry infill. Various models of finite element consisting of six and nine storey buildings were subjected to seismic loading by performing equivalent static analysis and response spectrum analysis as per IS code. By incorporating masonry infill in the model, model analysis predicted the dynamic behaviour of the structure. A significant increase in column shear and moment was experienced in the presence of infill panel. In addition to that, study suggested to design the columns of ground storey twice the magnitude of shear and moment calculated from linear static analysis.

2.3 Analytical & Experimental Studies on Shear wall

Lopes (2001) [12] described a comprehensive test in order to study the seismic performance of reinforced concrete walls subjected to extreme conditions and a shear failure was observed. To conduct this experiment, a test setup was designed to impose beam behaviour and low shear ratio was maintained during the test had been described in this work. Finally, observations were made and some special features described that were failure mode dependent.

Rana *et al.* (2004) [13] performed a nonlinear static analysis of a 19-storey reinforced concrete building with total area of 430,000 Sq ft. located in San Francisco. The building was typically designed as per 1997 Uniform Building Code with shear walls as a lateral resisting system to check the provisions and guidelines of the Life Safety performance level when subjected to design earthquake and results were presented in this work.

Lee *et al.* (2007) [14] studied the response of seismic parameters of three different models of 17-storey reinforced concrete wall building with various types of irregularity at the bottom storey when subjected to the same series of scaled earthquake motions. The first model consists of moment resisting frame symmetrical in nature and next model had an infill shear wall in the middle frame and last one, third had an infill shear wall provided only in exterior frames. On the basis of test observations, following conclusions were out forward and presented that the calculated fundamental time periods for other models than moment resisting frames and shear wall were found to be reasonable in UBC 97 and AIK 2000 . The total absorption of energy by damage was similar irrespective of the location and existence of the infill shear wall. The huge amount of energy absorption was due to overturning and finally followed due to shear deformation. The rigid system of upper storey rendered rocking behaviour of the lower frame. Therefore, the self weight of the structure contributed about 23% of resistance against the total turning moment.

Esmaili *et al.* (2008) [15] studied the structural aspects of a 56-storey reinforced concrete tall building located in highest seismic active area. For this structure, shear wall and irregular opening system was provided for lateral loads and gravity loads which might resulted some important issues in the behaviour of shear wall, coupling beams etc. For seismic assessment, numerous nonlinear analyses were used to evaluate its structural behaviour with prevailing retrofitting provisions as per FEMA 356. A study of assessment of the load bearing system with some special features had been considered and presented. At the end, a general assessment of ductility levels of shear wall was described in this work.

Fahjan *et al.* (2010) [16] throughly studied the various types of modelling approaches for modellling the linear and nonlinear behaviour of shear wall of buildings for structural analyses. Based on overall structural behaviour of the system, results of analyses using various modelling approaches were obtained and comapared.

Gonzales and Almansa (2012) [17] conducted a research work aiming to provide well grounded seismic provisions and guidelines for the design of thin wall structures especially buildings. The starting goals are to study the seismic behaviour of these structures and proposing initial criteria for design and spread the research to a great extent for future needs. This exploration concentrates on buildings situated in Peru, being illustrative of the circumstances in other nations. The vulnerability of these buildings was tested by nonlinear static and nonlinear dynamic analyses with structural characteristics were acquired from accessible testing data. The extracted results showed that seismic capacity was quite low of these buildings. However, minor corrections in the structural configuration may upgrade the seismic performance of such buildings. Inexpensive and effective design suggestions were issued.

Martinelli *et al.* (2013) [18] studied the capability of two distinctive of fiber beam-column finite elements to simulate the dynamic behaviour of a shear wall using shake table test.

Todut *et al.* (2014) [19] presented the results of an experimental program developed to study the seismic performance of precast reinforced concrete wall panels with and without openings. The specimen characteristics and reinforcement configuration were taken from a typical Romanian project used widely since 1981 and scaled 1:1.2 due to the constraints imposed by the laboratory facilities. This type of precast wall panels was used mostly for residential buildings with multiple flats built from 1981 to 1989. The performance and failure mode of all of the panels tested revealed a shear type of failure that is influenced by the opening type, and critical areas and lack of reinforcement were observed in certain regions. A numerical analysis was performed to create a model that could predict the behaviour of the precast reinforced concrete shear walls of different parameters.

Lu *et al.* (2015) [20] developed a new shear wall element model and associated material constitutive models based on the open source finite element (FE) code OpenSees, in order to perform nonlinear seismic analyses of high-rise RC frame-core tube structures. A series of shear walls, a 141.8 m frame-core tube building and a super-tall building (the Shanghai Tower, with a height of 632 m) are simulated. The rationality and reliability of the proposed element model and analysis method are validated through comparison with the available experimental data as well as the analytical results of a well validated commercial FE code. The research outcome will assist in providing a useful reference and an effective tool for further numerical analysis of the seismic behavior of tall and super-tall buildings.

2.4 Modelling for Nonlinear Behaviour

2.4.1 Mander *et al.* (1988) Concrete Model

A nonlinear uniaxial constant confinement model, initially follows the constitutive relationship given by Mander *et al.* (1988) [21] along with the cyclic rules proposed by Martinez-Rueda and Elnashai (1997) [22]. The effects of confinement of the lateral transverse reinforcement steel are implemented by following the rules proposed by Mander *et al.* (1998) and throughout the entire range of stress-strain, a constant confining pressure is assumed and shown here in Fig. 2.1. The following parameters are considered to define the mechanical characteristics of the concrete material are as follows.

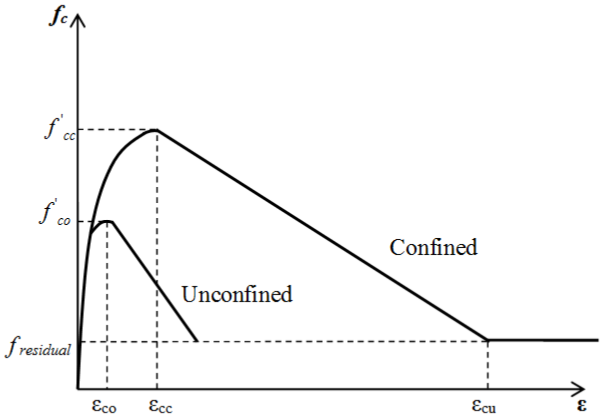

Figure 2.1: Stress-strain relationship of confined and unconfined concrete of Mander *et al.* (1998) .

1. **Compressive strength -** f_c

   The present value adopted for this parameter is 25 MPa.

2. **Tensile strength -** f_t

   The tensile stress capacity of the concrete material can be estimated by using this equation, $f_{cr} = 0.7\sqrt{f_{ck}}$ where f_{cr} represents the appropriate tensile strength, as suggested by IS 456. The present value is set to 3.5 MPa.

3. **Modulus of elasticity -** E_c

   An initial elastic stiffness of the material and easily extracted from the given equation, $E_c = 5000\sqrt{f_{ck}}$ where E_c represents the appropriate tensile strength, as suggested by IS 456. At present, a value of 25000 MPa is provided for this parameter.

4. **Strain at peak stress -** ε_c

   The strain related to the point of unconfined peak compressive stress (f_c). For this, a value of 0.002 mm/mm is assumed.

5. **Specific weight -** γ

   It defines the specific weight of the concrete material. The present value adopted for this parameter is set to 25 kN/m3.

2.4.2 Menegotto–Pinto Steel Model (1973)

A typical uniaxial steel model based on a simple, yet efficient, stress-strain relationship proposed by Menegotto and Pinto (1973) [23], coupled with the isotropic hardening rules proposed by Filippou *et al.* (1983) [24] as shown here in Fig. 2.2. These are the following parameters are adopted to describe the mechanical characteristics of the concrete material are given below:

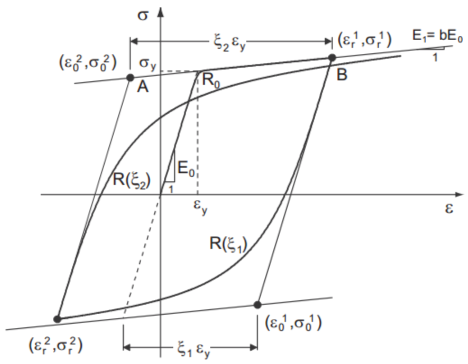

Figure 2.2: Stress-strain relationship of Menegotto–Pinto steel model (1973).

1. **Modulus of elasticity -** E_s

   The initial elastic stiffness of the reinforcement steel material is assigned to 200 GPa.

2. **Yield strength -** f_y

   The yielding stress of the present material is set 415 MPa.

3. **Strain hardening parameter -** μ

   The ratio between the post-yield stiffness (E_{sp}) and the initial elastic stiffness (E_s) of the material and calculated by using this equation, $E_{sp} = (f_{ult} - f_y)/(-f_y/E_s)$ where f_{ult} represents the ultimate or maximum stress and strain capacity of the material respectively. For this, a value of 0.004 is adopted to this parameter.

4. **Fracture/buckling strain -** E_{ult}

   The strain value used for this parameter is 0.1 at which fracture or buckling occurs .

5. **Specific weight -** γ

   At present, a value of 76.97 kN/m3 is provided as specific weight of the reinforcement steel.

2.4.3 Infill Wall Panel Element (Crisafulli, 1997)

A four-node masonry infill wall panel element, initially developed and programmed by Crisafulli (1997) [25] to capture the nonlinear response of masonry infill wall panels in frame structures as shown in Fig. 2.3. Here, each panel is represented by six strut members; each diagonal direction features two parallel struts to carry axial loads across two opposite diagonal corners and a third one to carry the shear from the top to the bottom of the panel. This latter strut only acts across the diagonal that is on compression, hence its "activation" depends on the deformation of the panel. The axial load struts use the masonry strut hysteresis model, while the shear strut uses a dedicated bilinear hysteresis rule and following are the required parameters for modelling of infill wall.

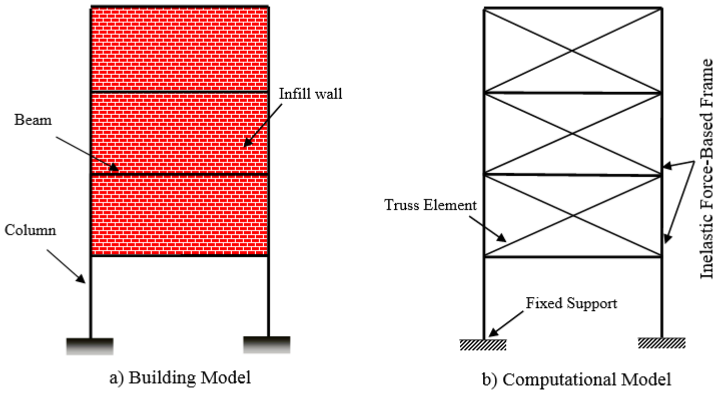

Figure 2.3: Typical building model and corresponding infill wall panel element (Crisafulli, 1997).

1. **Infill Panel Thickness - t**

   The value of brick infill wall thickness is 0.23 m provided here.

2. **Out-of-plane failure drift**

   The present value assigned to this parameter is set to 5% of the storey height.

3. **Equivalent contact length -** h_z

   The value implemented here is 23% of vertical panel side.

4. **Horizontal and Vertical offsets -** X_oi **and** Y_oi

   The horizontal offset is 2.4% and the vertical offset provided here is 10% of the horizontal and vertical panel side respectively.

5. **Proportion of stiffness assigned to shear -** γ_s

   A value of 20% has been used for this parameter.

6. **Specific weight -** γ

   A value of 18.85 kN/m3 is taken for this parameter.

Strut Curve Parameters The given parameters are considered to characterise the response curve.

1. **Initial Young modulus -** E_m

   The present values used here is 5000 MPa.

2. **Compressive strength -** $f_{m\theta}$

   The compressive strength adopted here is set to 9.09 MPa.

3. **Tensile strength -** f_t

   At present, the tensile strength is equal to zero.

4. **Strain at maximum stress -**ε_m

   Herein, the present value of 0.002 mm/mm is used.

5. **Ultimate strain -** ε_{ult}

   The value of ultimate strain is 0.0053 mm/mm for infill panel.

6. **Closing strain -** ε_{cl}

   Herein, a value of 0.004 mm/mm has been adopted to closing strain.

Shear Curve Parameters

These are the following parameters taken into account for the response curve.

1. **Shear bond strength -** τ_o

   The present value of shear bond strength is 0.45 MPa here.

2. **Friction coefficient -** μ

   A value of 0.3 is adopted at present.

3. **Maximum shear strength -** τ_{max}

   A value of 3.177 MPa is used for this parameter.

4. **Reduction shear factor -** α_s

   For this, a value of 1.5 is used herein.

2.4.4 Inelastic Force-Based Frame Element

A three-dimensional nonlinear force based beam-column element effective in modelling of space frames with material [26] and geometric nonlinearities [27]. For the present work, fiber approach is implemented to represent the nonlinear behaviour of the the cross-section, whereas, each fiber is shown here in Fig.

2.4 connected with a uniaxial stress-strain relationship following the fiber discretization of RC section. The sectional stress-strain state of beam-column elements is derived from the integration of the nonlinear uniaxial material response of the individual fibers in which the section has been fragmented, fully considering the spread of nonlinearity along the element length and across the section depth [28] [29].

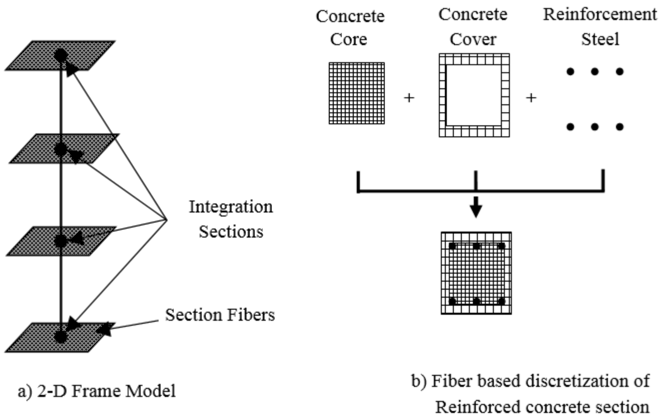

Figure 2.4: Inelastic Force–Based Frame Element.

2.5 Validation Study

2.5.1 Description

In order to validate the modelling and analysis approach for nonlinear analysis in the present study, the case studies of RC frames reported by Mondal *et al.* (2013) [1] is considered. The RC frames having four bays with number of storeys two, four and eight have been modelled in Seismostruct v7.0 using nonlinear force based frame element as shown in Fig. 2.4. The paper describes a full-scale, two, four and eight storey reinforced concrete frame models, two-dimensional typical symmetric in plan, a regular office building located in the seismic zone IV as per IS 1893(2002) with fundamental details are provided here in Table 2.1. The elevation of four-storey RC frame is shown in Fig. 2.5 and the structure has four bays of width 6 m in both direction having medium soil condition. The floor to floor height is 4.0 m for all the storeys and the depth of foundation is 3.0 m. The design base shear for all building frames is derived from the equation:

$$V_d = \frac{ZIS_a}{2Rg}W \tag{2.1}$$

where Z denotes the zone factor (= 0.24 for zone IV), I is the structures importance factor (= 1 for these buildings), R = 5.0 for ductile or special moment resisting frames (SMRF), S_a/g is the average response acceleration coefficient for rock soil or soil sites, and W is the seismic weight of the structure. A typical elevation of 4-storey frame is shown here. The RC frame is designed with M 25 grade concrete and Fe 415 grade reinforcements steel.

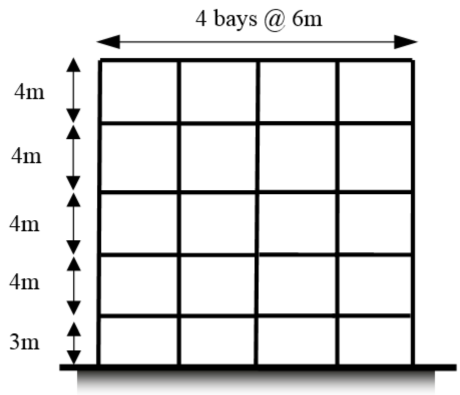

Figure 2.5: Four storey RC frame building.

Table 2.1: Fundamental details of the RC frames considered for the case study [1]

Frame	Height(m)	T_d(s)	W(kN)	Ah=V_d/W	V_d(kN)
2-Storey	11	0.453	4650	0.0600	279
4-Storey	19	0.683	7770	0.0478	371
8-Storey	35	1.08	13800	0.0302	416

2.5.2 Geometry & Modelling

All the necessary details of beams and columns are given here in Table 2.2 for the present work. For capturing nonlinear behaviour, Kent and Park model is employed for the modeling of beams and columns of the reinforced concrete frame. The lateral distribution of force for the nonlinear pushover analysis to compute R for each frame is suggested as per IS 1893 (2002).

Table 2.2: Cross-section details of RC Frames [1]

Frame	Floor	Member	Width(mm)	Depth(mm)	Reinforcement Details
2-Storey	Beam	1–2	250	500	3–25ϕ + 2–20ϕ (top) + 2–25ϕ + 1–20ϕ (bottom)
	Column	1–2	450	450	8–25ϕ (uniformly distributed)
4-Storey	Beam	1–4	300	600	6–25ϕ (top) + 3–25ϕ (bottom)
	Column	1–4	500	500	12–25ϕ (uniformly distributed)
8-Storey	Beam	1–4	300	600	6–25ϕ (top) + 3–25ϕ (bottom)
	Column	1–4	600	600	12–25ϕ (uniformly distributed)
	Beam	5–8	300	600	6–25ϕ (top) + 3–25ϕ (bottom)
	Column	5–8	500	500	12–25ϕ (uniformly distributed)

$$Q_i = V_d \frac{W h_i^2}{\sum\limits_{i=1}^{n} W h_i^2} \tag{2.2}$$

where Q_i is the equivalent lateral force on the i^{th} floor, W_i the seismic weight of the i^{th} floor, h_i the height up to the i^{th} floor, and n is the total number of storeys.

2.5.3 Comparison of Pushover Curves

The comparison of nonlinear pushover curves has been made here. From the analysis, it can be seen that the pushover curves are much closer to the results of published literature taken here for the validation of modelling approaches used for the present study. The comparison of pushover curves between the present study and published literature is shown hereafter in Fig. 2.6 to Fig. 2.8 for two, four and eight storey frame respectively.

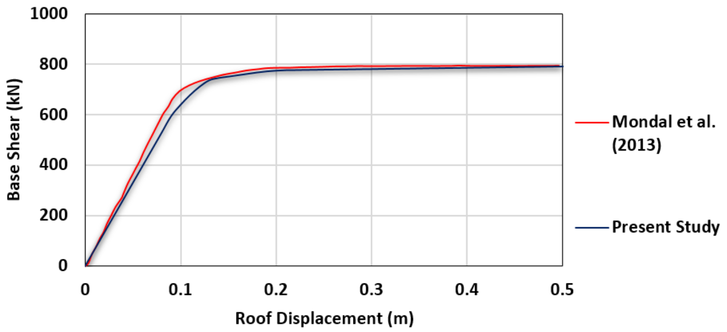

Figure 2.6: Pushover curves of the two-storey frame.

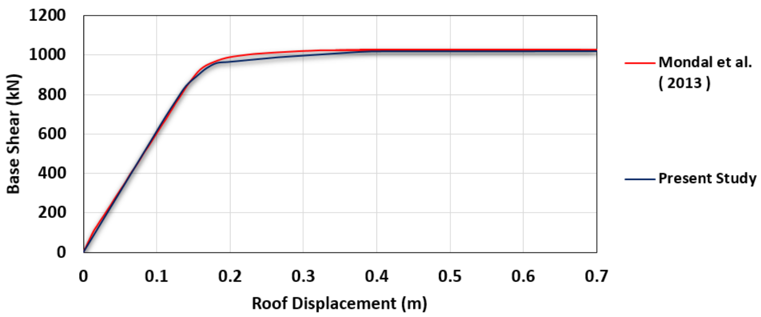

Figure 2.7: Pushover curves of the four-storey frame.

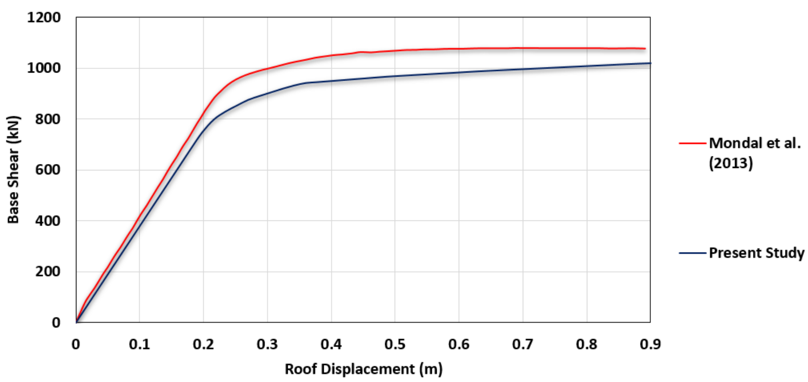

Figure 2.8: Pushover curves of the eight-storey frame.

2.6 Summary

The current chapter reviewed many published literature related to the open ground storey buildings and their seismic performance periodically. After reviewing, it is found that the research work is not significant and effective to deal with the poor performance of OGS buildings under seismic action. From the observation, it can be seen that the shear wall is very effective in providing stiffness and strength to this structure. Hence, a reinforced concrete shear wall is taken into account to improve the seismic performance of open ground storey building for the present study. At the end, the results of the validation study are fairly matched with the pushover curves of the present study. The next chapter examined a case study of four-storey OGS frames with reinforced concrete shear wall and applying various multiplication factors (MF).

Chapter 3

Open Ground Storey Building with RC Shear Wall

3.1 General

Details of the four storey OGS frame for the present study is presented in the first part of this chapter. The OGS frames are considered to be strengthened and re-designed using shear walls. In order to understand the comparative performance of OGS frames designed using various multiplication factors, they are also included in the group. Design details of all frames are provided in the first part of this Chapter. Second part of this Chapter provides the nonlinear modelling and performance criteria considered in the pushover analysis. Subsequently, it presents displacement profiles from linear static analyses. A comparison of pushover curves of all the frames and a comparative cost analysis of each frame is reported.

3.2 Design of 4 Storey RC Frames

3.2.1 Description

In the present study, a four storey reinforced concrete frame building has been considered in order to study the seismic performance with an application of reinforced concrete shear wall. The OGS frames also designed by various multiplication factors (MF) to the ground storey columns and/or first storey columns as per Indian Standard IS 1893 (2000) as shown in Fig. 3.1 to Fig. 3.3. The building is assumed to be symmetric in plan. Therefore, a single plane frame has been adopted to be illustrative of the building along one direction for the design and analysis. The design of the frames are carried out using ETABS 2013 [30].

3.2.2 Geometry of all example frames

All the frames considered in the present study are explained below.

1. The elevation of bare frame, B 1.0 is provided in Fig. 3.1a
2. The elevation of fully infilled frame, F 1.0 in which the infill wall is considered in all the storeys is provided in Fig. 3.1b
3. The elevation of OGS frame, O 1.0 in which the the multiplication factor of 1.0 is used for the design of ground storey columns is provided in Fig. 3.1c
4. The elevation of OGS frame strengthened with shear wall, OS 1.0 is provided in Fig. 3.1d
5. The elevation of OGS frame re-designed with IS 13920 (1993) incorporating shear wall, OSR 1.0 is provided in Fig. 3.1e
6. The elevation of OGS frames designed with MF of 1.5, 2.0, 2.5, 3.0 in the ground storey alone are shown in Figs.3.2a to 3.2d respectively.
7. The elevation of OGS frames designed with MF of 1.5, 2.0, 2.5, 3.0 in both the ground storey and first storey are shown in Figs.3.3a to 3.3d respectively.

The sectional details of columns and beams of all the frames are illustrated in Table 3.1 to 3.3.

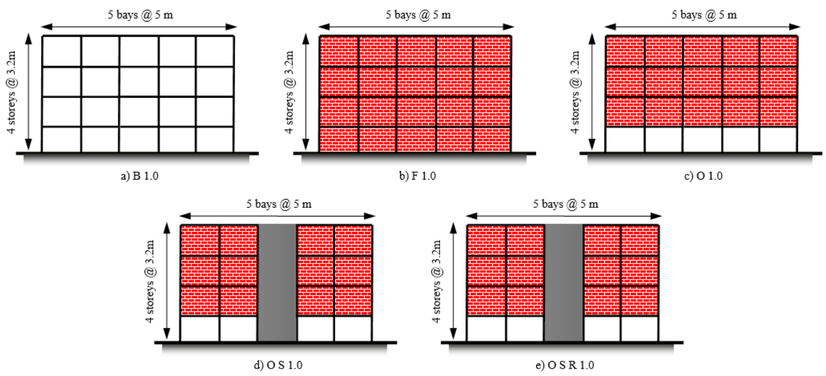

Figure 3.1: 4 Storey RC Frames with One Multiplication Factor (Fundamental Models).

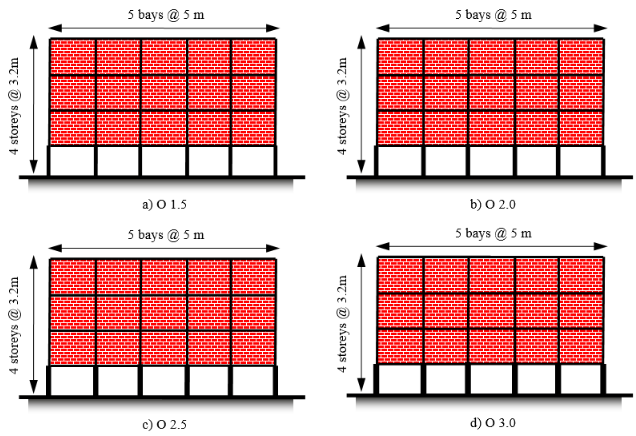

Figure 3.2: 4 Storey RC Frames with different Multiplication Factor for Ground Only.

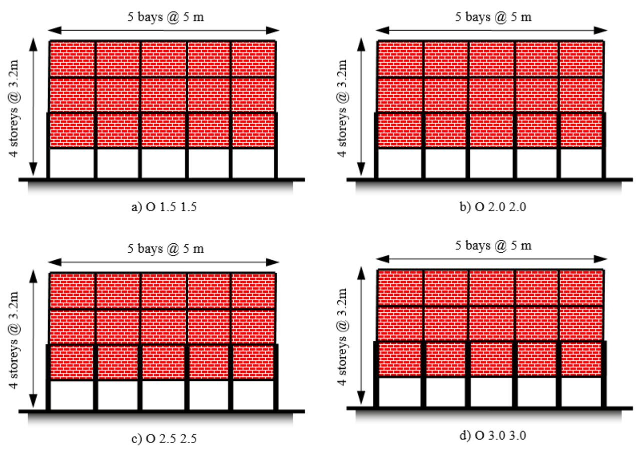

Figure 3.3: 4 Storey RC Frames with different Multiplication Factor for Ground and First Storey Only

3.2.3 Fundamental Time Periods of all frames

Eigen value analysis of computational models of all the frames is conducted to obtain the fundamental time periods. The time periods as per IS 1893 (2002) [31] are also calculated. The time periods of all the frames from both eigen value analysis and using empirical formula from code are tabulated in the Table 3.4.

Table 3.1: Fundamental Time Period of the Structures

Frame Storey	Computational		IS 1893 (2002)	
	Time period (sec)	Frequency (Hertz)	Time period (sec)	Frequency (Hertz)
B 1.0	0.330	3.02	0.507	19.7
O 1.0	0.316	3.15	-	-
F 1.0	0.080	12.45	0.230	4.34
O S 1.0	0.070	14.14	-	-
O S R 1.0	0.074	13.36	-	-
O 1.5	0.239	4.17	-	-
O 2.0	0.235	4.24	-	-
O 2.5	0.201	4.95	-	-
O 3.0	0.154	6.46	-	-
O 1.5,1.5	0.235	4.24	-	-
O 2.0,2.0	0.231	4.31	-	-
O 2.5,2.5	0.193	5.15	-	-
O 3.0,3.0	0.141	7.07	-	-

3.2.4 Design details of all the frames

A full scale four storey reinforced concrete frame with 5 numbers of bays of width as 5m and the height of the column is set to 3.2 m for the present work. Basically, all the moment frames are designed for the highest seismic active zone (zone V with PGA of 0.36g) as per IS 1893 (2002) having medium soil conditions (N-value

10 to 30). Herein, the building is designed with M 25 and Fe 415 as concrete and reinforcement steel material respectively. From the analysis point of view, all the gravity and lateral loads are considered with neglecting the stiffness and strength of masonry infill wall as per Indian Standard and for the design consideration of RC frame elements such as beams and columns, IS 456 (2000) [32] has been employed and detailed as per IS 13920 (1993) [33]. In order to study the effect of RC shear wall on seismic performance of reinforced concrete frame building, a reinforced concrete shear wall is designed and detailed as per the provisions and guidelines put forward by IS 456 (2000) and IS 13920 (1993).

In order to observe the effect of multiplication factor (MF) values on the performance of an open ground storey building under earthquake action, different MF values running from 1.5, 2.0, 2.5 and 3.0 are taken into account to design the ground storey columns and/or first storey columns of the OGS buildings as shown in Fig. 3.2 and Fig. 3.3 respectively.

For the sake of simplicity, some naming scheme is adopted to address all the frames considered in the present study like 'B'(Bare Frame), 'O'(Open Ground Storey) and 'F'(Fully Infilled Frames) and by applying different multiplication factor values to the different storeys of an open ground storey building, various designations are used to represent the MF values in the corresponding storeys to differentiate between each other OGS frame.

For example, O X,Y designates, an Open Ground Storey with MF used in the ground storey as 'X'and 'Y'for the first storey. For linear static analysis case, bare frame and open ground storey with RC shear wall frame without brick masonry infill wall with four load combinations are considered in order to account the maximum effect of gravity and lateral loads as defined in IS 1893 (2002). At present, C-1, C-2, C-3 and C-4 refers to 1.5(DL+IL), 1.2(DL+IL+EL), 1.5(DL+EL) and 0.9 DL+1.5 EL respectively where as DL stands for dead load, IL as imposed load and EL for earthquake load herein.

Table 3.2: Sections and Reinforcement details for Columns

Frame Configuration	Floor	Width(mm)	Depth(mm)	Reinforcement Details
B 1.0	1	350	350	8–20ϕ
	2–4	350	350	8–18ϕ
F 1.0	1	350	350	8–20ϕ
	2–4	350	350	8–18ϕ
O 1.0	1	350	350	8–20ϕ
	2–4	350	350	8–18ϕ
O S 1.0	1	350	350	8–20ϕ
	2–4	350	350	8–18ϕ
O S R 1.0	1–2	300	300	4–20ϕ
	3–4	300	300	4–20ϕ
O 1.5	1	425	425	8–22ϕ
	2–4	350	350	8–18ϕ
O 2.0	1	425	425	8–25ϕ
	2–4	350	350	8–18ϕ
O 2.5	1	475	475	12–25ϕ
	2–4	350	350	8–18ϕ
O 3.0	1	600	600	16–25ϕ
	2–4	350	350	8–18ϕ
O 1.5,1.5	1–2	425	425	8–22ϕ
	3–4	350	350	8–18ϕ
O 2.0,2.0	1–2	425	425	8–25ϕ
	3–4	350	350	8–18ϕ
O 2.5,2.5	1–2	475	475	12–25ϕ
	3–4	350	350	8–18ϕ
O 3.0,3.0	1–2	600	600	16–25ϕ
	3–4	350	350	8–18ϕ

Table 3.3: Sections and Reinforcement details for Beams with One Multiplication Factor

Frame Configuration	Floor	Width(mm)	Depth(mm)	Reinforcement Details Top — Bottom
B 1.0	1–2	300	375	5–20ϕ — 4–20ϕ
	3	300	375	4–20ϕ — 3–20ϕ
	4	300	325	4–20ϕ — 3–20ϕ
F 1.0	1–2	300	375	5–20ϕ — 4–20ϕ
	3	300	375	4–20ϕ — 3–20ϕ
	4	300	325	4–20ϕ — 3–20ϕ
O 1.0	1–2	300	375	5–20ϕ — 4–20ϕ
	3	300	375	4–20ϕ — 3–20ϕ
	4	300	325	4–20ϕ — 3–20ϕ
O S 1.0	1–2	300	375	5–20ϕ — 4–20ϕ
	3	300	375	4–20ϕ — 3–20ϕ
	4	300	325	4–20ϕ — 3–20ϕ
O S R 1.0	1–2	300	325	4–20ϕ — 3–20ϕ
	3	300	325	4–20ϕ — 3–20ϕ
	4	300	300	3–20ϕ — 2–20ϕ

Table 3.4: Sections and Reinforcement details for Beams with different Multiplication Factor for Ground and First Storey Only

Frame Configuration	Floor	Width(mm)	Depth(mm)	Reinforcement Details Top — Bottom
O 1.5	1–2	300	375	5–20ϕ — 4–20ϕ
	3	300	375	4–20ϕ — 3–20ϕ
	4	300	325	4–20ϕ — 3–20ϕ
O 2.0	1–2	300	375	5–20ϕ — 4–20ϕ
	3	300	375	4–20ϕ — 3–20ϕ
	4	300	325	4–20ϕ — 3–20ϕ
O 2.5	1–2	300	375	5–20ϕ — 4–20ϕ
	3	300	375	4–20ϕ — 3–20ϕ
	4	300	325	4–20ϕ — 3–20ϕ
O 3.0	1–2	300	375	5–20ϕ — 4–20ϕ
	3	300	375	4–20ϕ — 3–20ϕ
	4	300	325	4–20ϕ — 3–20ϕ
O 1.5,1.5	1–2	300	375	5–20ϕ — 4–20ϕ
	3	300	375	4–20ϕ — 3–20ϕ
	4	300	325	4–20ϕ — 3–20ϕ
O 2.0,2.0	1–2	300	375	5–20ϕ — 4–20ϕ
	3	300	375	4–20ϕ — 3–20ϕ
	4	300	325	4–20ϕ — 3–20ϕ
O 2.5,2.5	1–2	300	375	5–20ϕ — 4–20ϕ
	3	300	375	4–20ϕ — 3–20ϕ
	4	300	325	4–20ϕ — 3–20ϕ
O 3.0,3.0	1–2	300	375	5–20ϕ — 4–20ϕ
	3	300	375	4–20ϕ — 3–20ϕ
	4	300	325	4–20ϕ — 3–20ϕ

3.3 Effect of shear walls-Linear Static Analysis

In order study the effect of providing a shear wall to the bare frame a linear static analysis is conducted on a bare frame and bare frame with RC shear wall frame. The two frames are shown in Fig. 3.4 (a) and (b). The linear analysis is conducted on the two models and lateral displacements at each storey levels are computed. The lateral storey displacements are tabulated in Table 3.5. The storey displacements and storey drift obtained from analysis are plotted in Figs. 3.5 and 3.6 respectively. Following are the observations from the linear static analysis.

1. It can be seen that RC shear wall reduces the maximum lateral sway from 28.6 mm in B 1.0 to 1.9 mm in OS 1.0, and the percentage reduction is about 93%.

2. The reduction in the maximum storey drift in the bare frame is about 95% after the addition of shear walls

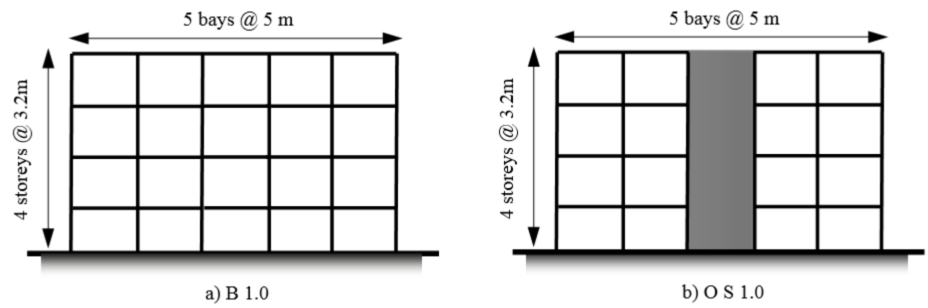

Figure 3.4: Bare frame with and without shear wall

Table 3.5: Maximum Lateral Storey Displacement - Linear static analysis

Frame	B 1.0				O S 1.0			
Storey	C-1	C-2	C-3	C-4	C-1	C-2	C-3	C-4
Base	0	0	0	0	0	0	0	0
1	0	5.3	6.6	6.6	0	0.2	0.2	0.2
2	0	12.7	15.8	15.8	0	0.6	0.7	0.7
3	0	18.9	23.7	23.7	0	1.1	1.3	1.3
4	0	22.9	28.6	28.6	0	1.6	1.9	1.9

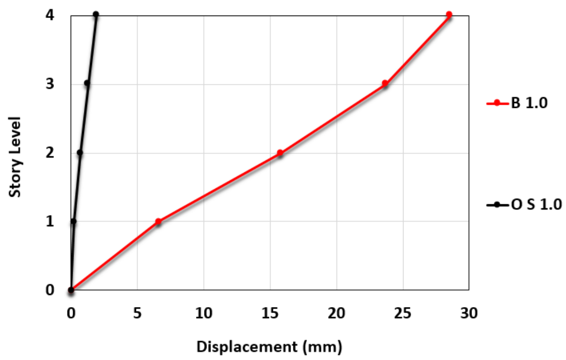

Figure 3.5: Storey displacements- Linear Static Analysis

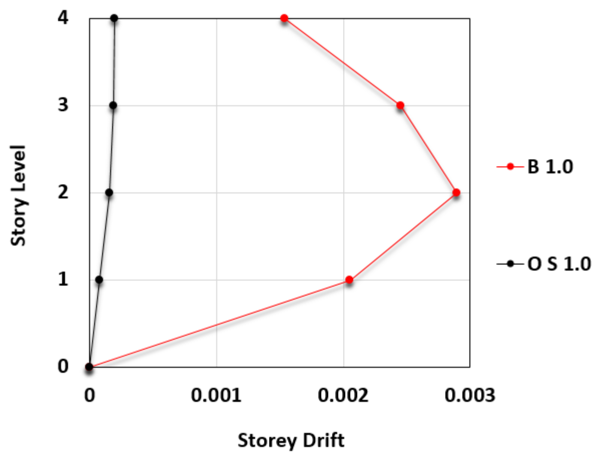

Figure 3.6: Maximum storey drift of the 4-storey frames.

3.4 Nonlinear Modelling

All the example frames are modelled in the program Seismosoft v7.0. [34].

1. The Mander*et al.* (1988) [21] concrete confinement model is used for concrete sections.

2. The Menegotto-Pinto steel model (1973) [23] is considered for defining the reinforcement steel.

3. Crisafulli Inelastic infill panel (1997) [25] is adopted for modelling of Infill walls through a four-node masonry panel element.

4. Columns, beams and walls are modelled by using inelastic force-based inelastic frame elements with 4 numbers of integration sections.

3.5 Loading

In nonlinear static analysis (Pushover analysis), the load applied to the structure consists of permanent gravity loads in the vertical (z) direction and lateral loads in horizontal (x) direction. Herein, the permanent gravity load consists of dead load of the slab (5 m x 5 m panel) including floor finishes is taken as 3.75 kN/m^2 and live load as 3 kN/m^2. The lateral load is applied as incremental displacement controlled procedure till the attainment of the target displacement at the controlled node. The analysis completes when the target displacement is reached or when structural or numerical collapse occurs. The distribution of lateral forces applied to the structure is uniform in nature and the value of target displacement has been used for this analysis is equivalent to 5 times of 0.004 times the storey height as per IS 1893 (2002) for storey drift limitation.

3.6 Performance Criteria

In the era of performance-based engineering, it is foremost that investigators and engineers are equipped with identifying the instants at which different performance limit states (e.g. non-structural damage, structural damage, collapse) will be reached. The following Performance criteria are defined for the present work.

1. **Performance limit (PL 1)** - Yielding of steel is identified by checking for (positive) steel strains larger than the ratio between yield strength and modulus of elasticity of the steel material. A value of 0.0038 has been assigned for this parameter and given in Table 3.8 here.

2. **Performance limit (PL 2)** - Spalling of cover concrete can be monitored by checking for (negative) cover concrete strains larger than the ultimate crushing strain of unconfined concrete material. At present, a value of -0.005 is taken and provided in Table 3.9 for this parameter.

3. **Performance limit (PL 3)** - Crushing of core concrete can be confirmed by checking for (negative) core concrete strains larger than the ultimate crushing strain of confined concrete material. Herein, a present value of -0.02 is adopted and presented in Table 3.10 for this parameter.

3.7 Performance Comparison of all the frames-Nonlinear static analysis

Nonlinear pushover analysis of all the frames are conducted to obtain the capacities of base shear and lateral displacement.

3.7.1 All the frames

Pushover curves of all the frames are shown in Fig. 3.7. It can be seen that the base shear capacity of the frames OS 1.0, OSR 1.0 and F 1.0 are significantly higher than B 1.0 and O 1.0. The initial stiffness of frames OS 1.0, OSR 1.0 and F 1.0 are also higher than that of remaining frames.

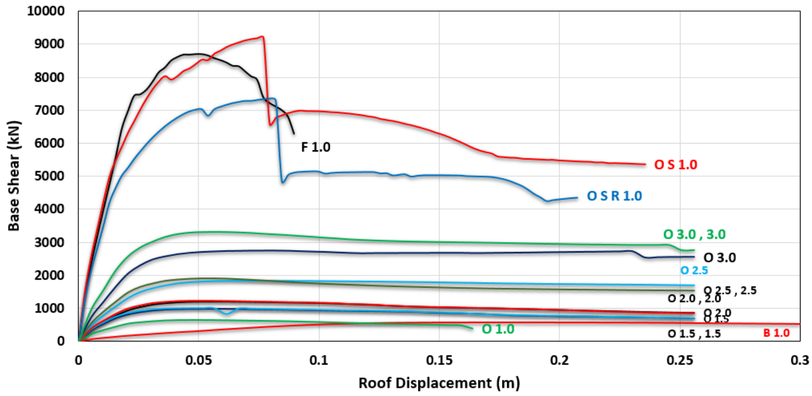

Figure 3.7: Pushover curves of all the 4 Storey Frames.

3.7.2 Frames with MF = 1

Pushover curves of all the frames with MF equal to unity are shown in Fig. 3.8 indicating the performance limits PL1, PL2 and PL3. It can be seen that the base shear capacity of frames OS 1.0, OSR 1.0 and F 1.0 are higher than B 1.0 and O 1.0 at each performance levels.

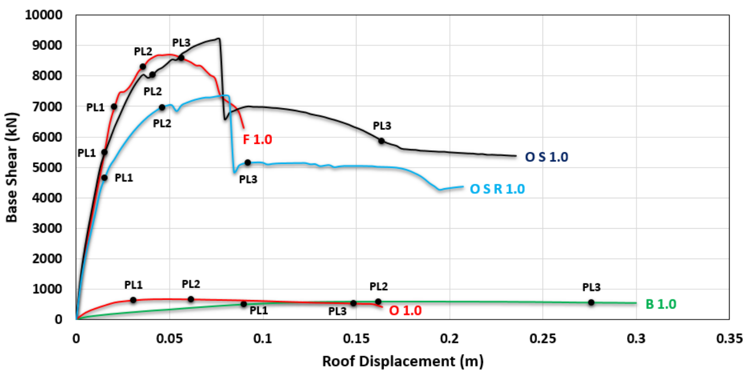

Figure 3.8: Pushover curves of the 4 Storey Frames with One Multiplication Factor.

3.7.3 Frames with MF applied in ground storey

Pushover curves of all the frames with MF applied in the ground storey columns are shown in Fig. 3.9 indicating the performance limits PL1, PL2 and PL3. As the MF values applied in the ground storey increases the base shear capacities also increases.

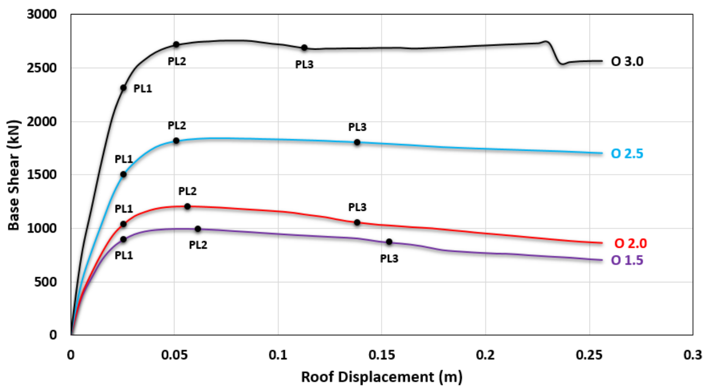

Figure 3.9: Pushover curves of the 4 Storey Frames with different Multiplication Factor for Ground Storey Only.

3.7.4 Frames with MF applied in both ground storey and first storey

Pushover curves of all the frames with MF applied in both the storeys are shown in Fig. 3.10 indicating the performance limits PL1, PL2 and PL3. As the MF value increases the base shear capacities also increases.

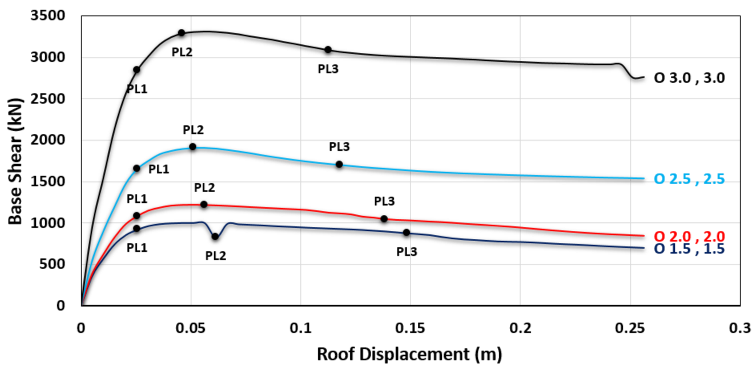

Figure 3.10: Pushover curves of the 4 Storey Frames with different Multiplication Factor for Ground and First Storey Only.

3.7.5 Maximum base shear capacity and lateral displacement capacity

The maximum base shear capacity and lateral displacement capacity of all the four storey frame models obtained from pushover analysis are tabulated in Table 3.6. Following are observations based on the maximum capacities.

1. After providing reinforced concrete shear wall to the open ground storey frame, the base shear capacity significantly increases by 92.85 % along with that lateral displacement capacity increases by 39.74 % under seismic loading.

2. With the application of various multiplication factor to the ground storey of an open ground storey frame ranging from 1.5 to 3.0 with an interval of 0.5 increment. The base shear capacity gradually increases from 34.15 % to 76.15% and the corresponding lateral displacement capacity improves from 9.80 % to 43.90 % respectively in a regular manner respectively.

3. By employing distinct multiplication factor to the ground and first storey both of an open ground storey frame running from 1.5,1.5 to 3.0,3.0 with an interval of 0.5,0.5 increment each. The base shear capacity gradually increases from 34.49 % to 80.14% relatively the lateral displacement capacity improves from 2.12 % to 17.85 % respectively in a regular manner.

4. In the course of present work, a bare frame has the lowest base shear capacity around 11.47 % and the highest lateral displacement capacity of 74.45 % with respect to open ground storey frame.

5. At the end, a full masonry infill wall frame has the base shear capacity of 92.45 % almost equivalent to the base shear capacity of the RC shear wall with the lateral displacement capacity of 4.17 % in opposite to an open ground storey frame.

6. While keeping constant the cross- section and reinforcement steel of shear wall and without altering the details of masonry infill wall and after slightly

minimizing the cross-section of the beams and columns of an open ground storey frame, the base shear capacity effectively increases by 91.06 % and the corresponding lateral displacement capacity increases by 41.77 % when subjected to earthquake loads.

Table 3.6: Maximum Base Shear Capacity and Lateral Displacement Capacity

Frame Model	Base Shear (kN)	% increase in Base Shear Capacity	Roof Displacement (m)	% increase in Roof Displacement Capacity
B 1.0	581.75	11.47	0.180	74.45
O 1.0	657.13	–	0.046	–
F 1.0	8708.71	92.45	0.048	4.17
O S 1.0	9201.03	92.85	0.076	39.47
O S R 1.0	7354.36	91.06	0.079	41.77
O 1.5	997.87	34.15	0.051	9.80
O 2.0	1203.44	45.39	0.056	17.85
O 2.5	1843.96	64.36	0.072	36.12
O 3.0	2756.05	76.15	0.082	43.90
O 1.5,1.5	1003.18	34.49	0.046	2.12
O 2.0,2.0	1217.06	46.00	0.051	9.80
O 2.5,2.5	1910.44	65.60	0.056	17.85
O 3.0,3.0	3308.70	80.14	0.056	17.85

3.7.6 Comparison of capacities at Performance limit PL 1

The comparison of capacities of all the frames are carried out with reference to tha capacities at PL1 performance level of open ground storey frame, O 1.0. The reference values of base shear is about 618.78 kN and the corresponding lateral displacement is obtained as 0.031 m.

1. The value of base shear at which first yielding of reinforcement steel reached, is increased by 88.73 % with a decrement of 51.62 % in lateral displacement of an open ground storey frame with RC shear wall under seismic loading.

2. By applying various multiplication factor to the ground storey of an open ground storey frame ranging from 1.5 to 3.0 with an interval of 0.5 increment. The value of base shear at which first yielding of reinforcement steel identified, is increased by from 31.05 % to 73.23 % while maintaining the constant value of lateral displacement of 19.35 % for each and every case respectively.

3. By applying various multiplication factor to the ground and first storey both of an open ground storey frame ranging from 1.5,1.5 to 3.0,3.0 with a period of 0.5,0.5 each increment. The value of base shear at which first yielding of reinforcement steel established, is improved by from 32.82 % to 78.20 % without showing any change in the value of lateral displacement of 19.35 % for each and every case respectively.

4. In bare frame, the first yielding of reinforcement steel is going to take at the lowest base shear of 486.56 kN normally less than 21.36 % with a significant increment in lateral displacement of 65.55 % against the open ground storey frame.

5. A full masonry infill wall frame has the huge base shear capacity of 91.14 % with reducing lateral displacement capacity of 32.25 % in comparison to the open ground storey frame at the first yielding of reinforcement steel.

6. By considering redesigned frame with shear wall, the value of base shear at which first yielding of reinforcement steel will happen, is increased by 86.70 % while reducing 51.62 % in lateral displacement capacity exactly equivalent to open ground storey frame with RC shear wall.

Table 3.7: Comparison of capacities at Performance limit PL 1

Frame Model	Base Shear (kN)	% increase in Base Shear Capacity	Roof Displacement (m)	% decrease in Roof Displacement Capacity
B 1.0	486.56	21.36	0.090	65.55
O 1.0	618.78	–	0.031	–
F 1.0	6983.77	91.14	0.021	32.25
O S 1.0	5492.07	88.73	0.015	51.62
O S R 1.0	4655.54	86.70	0.015	51.62
O 1.5	897.37	31.05	0.025	19.35
O 2.0	1037.96	40.38	0.025	19.35
O 2.5	1503.83	58.85	0.025	19.35
O 3.0	2311.60	73.23	0.025	19.35
O 1.5,1.5	921.15	32.82	0.025	19.35
O 2.0,2.0	1078.70	42.64	0.025	19.35
O 2.5,2.5	1654.84	62.60	0.025	19.35
O 3.0,3.0	2838.55	78.20	0.025	19.35

3.7.7 Comparison of capacities at Performance limit PL 2

The comparison of capacities of all the frames are carried out with reference to tha capacities at PL1 performance level of open ground storey frame, O 1.0. The reference values of base shear is about 648.75 kN and the corresponding lateral displacement is obtained as 0.061 m.

1. The value of base shear at which first spalling of unconfined concrete reached, is increased by 91.92 % with a decrement of 32.78 % in lateral displacement of an open ground storey frame with RC shear wall.

2. With various multiplication factor to the ground storey of an open ground storey frame ranging from 1.5 to 3.0 with an interval of 0.5 increment. The value of base shear at which first spalling of unconfined concrete verified, is increased by from 34.80 % to 76.10 % and the corresponding lateral displacement capacity initially matches with the open ground storey frame defined above and then slightly reducing by 16.39 % respectively.

3. By employing distinct multiplication factor to the ground and first storey both of an open ground storey frame running from 1.5,1.5 to 3.0,3.0 with an interval of 0.5,0.5 increment each. When first spalling of unconfined concrete recognized, the base shear capacity gradually increases from 21.46 % to 80.25% relatively the lateral displacement capacity initially coincides with the open ground storey frame defined above and then slightly reducing continuously by 24.59 % respectively.

4. In bare frame, the first spalling of unconfined concrete is going to take at the base shear of 580.62 kN generally less than 10.50 % with a significant increment in lateral displacement of 62.35 % against the open ground storey frame.

5. A full masonry infill wall frame has the huge base shear capacity of 92.18 % with reducing lateral displacement capacity of 42.62 % in comparison to the open ground storey frame at the first palling of unconfined concrete.

6. By considering redesigned frame with shear wall, the value of base shear at which first palling of unconfined concrete occurred, is increased by 90.68 % while reducing 24.59 % in lateral displacement capacity.

Table 3.8: Comparison of capacities at Performance limit PL 2

Frame Model	Base Shear (kN)	% increase in Base Shear Capacity	Roof Displacement (m)	% decrease in Roof Displacement Capacity
B 1.0	580.62	10.52	0.162	62.35
O 1.0	648.75	–	0.061	–
F 1.0	8302.88	92.18	0.035	42.62
O S 1.0	8035.97	91.92	0.041	32.78
O S R 1.0	6967.26	90.68	0.046	24.59
O 1.5	995.08	34.80	0.061	0.00
O 2.0	1203.44	46.09	0.056	8.19
O 2.5	1817.07	64.30	0.051	16.39
O 3.0	2714.68	76.10	0.051	16.39
O 1.5,1.5	826.11	21.46	0.061	0.00
O 2.0,2.0	1213.95	46.55	0.056	8.19
O 2.5,2.5	1909.94	66.03	0.051	16.39
O 3.0,3.0	3284.37	80.25	0.046	24.59

3.7.8 Comparison of capacities at Performance limit PL 3

The comparison of capacities of all the frames are carried out with reference to tha capacities at PL1 performance level of open ground storey frame, O 1.0. The reference values of base shear is about 512.99 kN and the corresponding lateral displacement is obtained as 0.148 m.

1. The value of base shear at which first crushing of confined concrete detected, is increased by 91.25 % with an increment of 9.75 % in lateral displacement

capacity of an open ground storey frame with RC shear wall.

2. With various multiplication factor to the ground storey of an open ground storey frame ranging from 1.5 to 3.0 with an interval of 0.5 increment. The value of base shear at which first crushing of confined concrete verified, is increased by from 40.95 % to 80.89 % and the corresponding lateral displacement capacity initially increases by 3.89 % and thereafter continuously reducing upto 23.64 % respectively.

3. By employing distinct multiplication factor to the ground and first storey both of an open ground storey frame running from 1.5,1.5 to 3.0,3.0 with an interval of 0.5,0.5 increment each. When first first crushing of confined concrete recognized, the base shear capacity gradually increases from 41.56 % to 83.35 % relatively the lateral displacement capacity initially coincides with the open ground storey frame defined above and then continuously reducing upto 23.64 % respectively.

4. In bare frame, the first crushing of confined concrete is going to take at the base shear of 547.63 kN slightly less than 6.32 % with a significant increment in lateral displacement of 46.37 % against the open ground storey frame.

5. A full masonry infill wall frame has the huge base shear capacity of 94.02 % with reducing lateral displacement capacity of 62.16 % in comparison to the open ground storey frame at the first crushing of confined concrete.

6. By considering redesigned frame with shear wall, the value of base shear at which first crushing of confined concrete occurred, is increased by 90.02 % while reducing 37.83 % in lateral displacement capacity.

Table 3.9: Comparison of capacities at Performance limit PL 3

Frame Model	Base Shear (kN)	% increase in Base Shear Capacity	Roof Displacement (m)	% decrease/increase in Roof Displacement Capacity
B 1.0	547.63	6.32	0.276	+46.37
O 1.0	512.99	–	0.148	–
F 1.0	8587.90	94.02	0.056	-62.16
O S 1.0	5862.53	91.25	0.164	+9.75
O S R 1.0	5135.87	90.02	0.092	-37.83
O 1.5	868.75	40.95	0.154	+3.89
O 2.0	1053.54	51.30	0.138	-6.75
O 2.5	1805.82	71.59	0.138	-6.75
O 3.0	2684.65	80.89	0.113	-23.64
O 1.5,1.5	877.95	41.56	0.148	0.00
O 2.0,2.0	1043.83	50.85	0.138	-6 75
O 2.5,2.5	1703.84	69.89	0.117	-20.94
O 3.0,3.0	3082.38	83.35	0.113	-23.64

3.8 Cost analysis

Material and labour cost for each frames are computed. A ratio of maximum base shear to total cost for each frame is calculated. Details of the cost for each frame is provided in the Table 3.10. Ratios of maximum base shear to total cost for each frame are presented in Fig. 3.12 to find out effective and economical frame under earthquake action. It can be seen that the ratio of more for F 1.0, which means that this frame is more economical. However, this frame may not serve the purpose of parking right in the ground storey of the building. Out of all other frames that may provide the parking space in the ground storey, OS 1.0 and OSR 1.0 are the most economical frames. From the cost analysis, it is found that the maximum base shear to cost ratio of OGS frames strengthened with shear wall is about 9 times more than that of OGS frame. In case of OGS frame re-designed with shear wall, the ratio is about 8 times more than that of OGS frame. After strengthening Open Ground storey buildings by applying various schemes of multiplication factors in line with the approach proposed by IS 1893 (2002), the maximum base shear to cost ratio is only about 3 times than that of OGS frames.

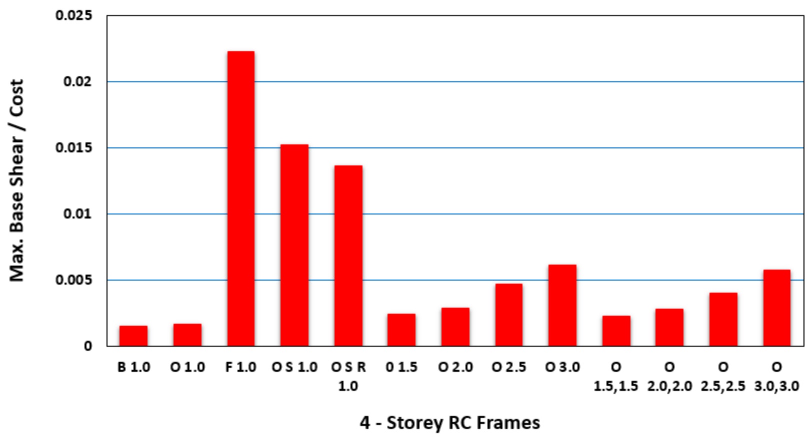

Figure 3.11: Comparison of Maximum base shear to cost ratio of all frames

Table 3.10: Cost analysis of RC frames with Shear wall and various MF

Frame	Concrete (cu m)		Reinforcement Steel (Kg)		Total cost (Rs.)
Storey	Column	Beam	Column	Beam	
B 1.0	9.408	10.11	1846.32	1587.27	389846
O 1.0	9.408	10.11	1846.32	1587.27	389846
F 1.0	9.408	10.11	1846.32	1587.27	389846
O S 1.0	9.408 + 11.904	8.088	4182.48	1269.82	608598
O S R 1.0	6.912 + 11.904	7.188	3692.64	1128.52	538196
O 1.5	10.524	10.065	2065.33	1580.20	413033
O 2.0	10.524	10.065	2065.33	1580.20	413033
O 2.5	11.388	8.004	2234.89	1256.62	393437
O 3.0	13.968	7.99	2741.22	1254.43	448709
O 1.5,1.5	11.640	10.02	2284.35	1573.14	436219
O 2.0,2.0	11.640	10.02	2284.35	1573.14	436219
O 2.5,2.5	13.368	9.97	2623.47	1565.29	472486
O 3.0,3.0	18.258	9.83	3583.13	1543.31	575144

Chapter 4

Summary & Conclusion

4.1 Summary

The objective of the present study was to compare the performance of OGS buildings strengthened with shear walls. Two different cases of providing shear wall in the OGS frames are considered. In the first case the shear walls are provided as a strengthening option whereas in the second case, the OGS frame is re-designed with shear wall. The OGS frames with Shear walls are considered as strengthening schemes in Four storey OGS frames and nonlinear static pushover analyses are conducted. In addition to that, the study includes the OGS frames with the ground storey columns strengthened by different schemes of multiplication factors. Following major conclusions are made with reference to the nonlinear static pushover curves obtained for all the example frames.

4.2 Major Conclusions

The following are the major conclusions from the present study

4.2.1 OGS frames strengthened with shear wall

1. The maximum capacities of base shear and roof displacement of the OGS frame strengthened with shear wall is increased by about 93% and 40%

respectively.

2. The maximum capacities of base shear and roof displacement of the OGS frame strengthened with shear wall is increased by about 5% and 37% respectively compared to a RC frame infilled in all storeys.

4.2.2 OGS frames re-designed with shear wall

1. The maximum capacities of base shear and roof displacement of the OGS frame re-designed with shear wall is increased by about 91% and 42% respectively.

2. The maximum base shear capacity of the OGS frame re-designed with shear wall is decreased by about 16% and the displacement capacity is increased by about 39% compared to a RC frame infilled in all storeys.

3. The maximum base shear capacity of the OGS frame re-designed with shear wall is decreased by about 20% and the displacement capacity is increased by about 4% compared to an OGS frames re-designed with shear wall.

4.2.3 Cost Analysis

1. The maximum base shear to cost analysis ratio for OGS frames strengthened with shear wall is more by about 9 times that of OGS frame.

2. The maximum base shear to cost analysis ratio for OGS frames re-designed with shear wall is more by about 8 times that of OGS frame.

3. The strengthening schemes in line with IS code procedure of applying multiplication factor could achieve only a maximum base shear to cost ratio of 3 times that of OGS frames.

Bibliography

[1] Apurba Mondal, Siddhartha Ghosh, and GR Reddy. Performance-based evaluation of the response reduction factor for ductile rc frames. *Engineering structures*, 56:1808–1819, 2013.

[2] CVR Murty. *IITK-BMTPC Earthquake Tips: Learning Earthquake Design and Construction*. National Information Centre of Earthquake Engineering, Indian Institute of Technology Kanpur, 2005.

[3] Bungale S Taranath. *Reinforced concrete design of tall buildings*. CRC Press, 2009.

[4] CVR Murty, Rupen Goswami, AR Vijayanarayanan, and Vipul V Mehta. Some concepts in earthquake behaviour of buildings. *Gujarat State Disaster Management Authority, Government of Gujarat*, 2012.

[5] CVR Murty and Sudhir K Jain. Beneficial influence of masonry infill walls on seismic performance of rc frame buildings. In *12th World Conference on Earthquake Engineering (Auckland, New Zealand, January 30)*, 2000.

[6] Robin Davis, Praseetha Krishnan, Devdas Menon, and A Meher Prasad. Effect of infill stiffness on seismic performance of multi-storey rc framed buildings in india. In *13 th World Conference on Earthquake Engineering. Vancouver, BC, Canada*, 2004.

[7] Hemant B Kaushik, Durgesh C Rai, and Sudhir K Jain. Stress-strain characteristics of clay brick masonry under uniaxial compression. *Journal of materials in Civil Engineering*, 19(9):728–739, 2007.

[8] Santiago Pujol, Amadeo Benavent-Climent, Mario E Rodriguez, and J Paul Smith-Pardo. Masonry infill walls: an effective alternative for seismic strengthening of low-rise reinforced concrete building structures. In *14th World Conference on Earthquake Engineering (Beijing, China, Unknown Month October 12)*, 2008.

[9] GV Mulgund and AB Kulkarni. Seismic assessment of rc frame buildings with brick masonry infills. *International Journal of advanced engineering sciences and technologies*, 2(2):140–147, 2011.

[10] J Prakashvel, C UmaRani, K Muthumani, and N Gopalakrishnan. Earthquake response of reinforced concrete frame with open ground storey. *Bonfring International Journal of Industrial Engineering and Management Science*, 2(4):91–101, 2012.

[11] N Sivakumar, S Karthik, S Thangaraj, S Saravanan, and CK Shidhardhan. Seismic vulnerability of open ground floor columns in multi storey buildings. *International Journal of Scientific Engineering and Research (IJSER)*, 2013.

[12] MS Lopes. Experimental shear-dominated response of rc walls: Part i: Objectives, methodology and results. *Engineering Structures*, 23(3):229–239, 2001.

[13] Rahul Rana, Limin Jin, and Atila Zekioglu. Pushover analysis of a 19 story concrete shear wall building. In *13th World Conference on Earthquake Engineering Vancouver, BC, Canada August 1-6, 2004 Paper No*, volume 133, 2004.

[14] Han-Seon Lee and Dong-Woo Ko. Seismic response characteristics of high-rise rc wall buildings having different irregularities in lower stories. *Engineering structures*, 29(11):3149–3167, 2007.

[15] O Esmaili, S Epackachi, M Samadzad, and SR Mirghaderi. Study of structural rc shear wall system in a 56-story rc tall building. In *The 14th world conference earthquake engineering*, 2008.

[16] YM Fahjan, J Kubin, and MT Tan. Nonlinear analysis methods for reinforced concrete buildings with shear walls. *14th econference in Earthquake Engineering*, 2010.

[17] H Gonzales and Francisco López-Almansa. Seismic performance of buildings with thin rc bearing walls. *Engineering Structures*, 34:244–258, 2012.

[18] Luca Martinelli, Paolo Martinelli, and Maria Gabriella Mulas. Performance of fiber beam–column elements in the seismic analysis of a lightly reinforced shear wall. *Engineering Structures*, 49:345–359, 2013.

[19] C Todut, D Dan, and V Stoian. Theoretical and experimental study on precast reinforced concrete wall panels subjected to shear force. *Engineering Structures*, 80:323–338, 2014.

[20] Xinzheng Lu, Linlin Xie, Hong Guan, Yuli Huang, and Xiao Lu. A shear wall element for nonlinear seismic analysis of super-tall buildings using opensees. *Finite Elements in Analysis and Design*, 98:14–25, 2015.

[21] John B Mander, Michael JN Priestley, and R Park. Theoretical stress-strain model for confined concrete. *Journal of structural engineering*, 114(8):1804–1826, 1988.

[22] J Enrique Martínez-Rueda and AS Elnashai. Confined concrete model under cyclic load. *Materials and Structures*, 30(3):139–147, 1997.

[23] M Menegotto. Pinto,(1973), pe, method of analysis for cyclically loaded reinforced concrete plane frames including changes in geometry and non-elastic behavior of elements under combined normal force and bending. In *IABSE Symposium on the Resistance and Ultimate Deformability of Structures Acted on by Well-Defined Repeated Loads, Lisbon*, 1973.

[24] Filip C Filippou, Egor Paul Popov, and Vitelmo Victorio Bertero. Effects of bond deterioration on hysteretic behavior of reinforced concrete joints. 1983.

[25] Francisco Javier Crisafulli. Seismic behaviour of reinforced concrete structures with masonry infills. 1997.

[26] Michalis Fragiadakis and Manolis Papadrakakis. Modeling, analysis and reliability of seismically excited structures: computational issues. *International Journal of Computational Methods*, 5(04):483–511, 2008.

[27] António A Correia and Francisco BE Virtuoso. Nonlinear analysis of space frames. In *III European Conference on Computational Mechanics*, pages 107–107. Springer, 2006.

[28] E Spacone, V Ciampi, and FC Filippou. Mixed formulation of nonlinear beam finite element. *Computers & Structures*, 58(1):71–83, 1996.

[29] Ansgar Neuenhofer and Filip C Filippou. Evaluation of nonlinear frame finite-element models. *Journal of Structural Engineering*, 123(7):958–966, 1997.

[30] ETABS Version. 13.1.4, 2013. *Computers and Structures, Inc., Berkeley, California*.

[31] Indian Standard. Criteria for earthquake resistant design of structures. *Indian Standards Institution, New Delhi, IS 1893 (Part 1)*, 2002.

[32] IS Code. Code 456-2000 code of practice for plain and reinforced concrete bureau of indian standards. *New Delhi*.

[33] IS BIS. 13920 ductile detailing of reinforced concrete structures subjected to seismic forces–code of practice. *New Delhi (India): Bureau of Indian Standards*, 1993.

[34] SeismoSoft. Seismostruct v7.0.0 - a computer program for static and dynamic nonlinear analysis of framed structures. 2014.